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Numerical model for predicting the structural response of composite UHPC–concrete members considering the bond strength at the interface

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Abstract

In this study, an improved finite element (FE) model was developed for the prediction of the structural behaviour of reinforced concrete members strengthened with ultrahigh-performance concrete (UHPC). A concrete damage model and an implicit solver in LS-DYNA were adopted in the numerical simulation. The model was calibrated and validated using experimental data. Accurately representing the interfacial bond characteristics of composite UHPC–concrete members was the primary challenge in developing the modelling technique. A novel technique using equivalent beam elements at the interface between UHPC and normal strength concrete (NSC) substrate was proposed for this purpose. The material properties of the equivalent beam elements were defined to represent the equivalent bond characteristics of NSC. The developed FE model was found to be able to effectively and efficiently predict the structural response of composite UHPC–concrete members with good accuracy.

Keywords: Ultrahigh-performance concrete (UHPC); UHPC strengthening; Interfacial bond strength; Finite element modelling; LS-DYNA

1. Introduction

Strengthening reinforced concrete (RC) structures with ultrahigh-performance concrete (UHPC) is an emerging technique for the design and protection of new or existing structures because of the superior mechanical properties of UHPC, including its high strength, low permeability, and energy absorption

[1]. In addition, UHPC has shown high bond strength and good adherence to normal-strength concrete (NSC) substrates [2–4]. Over the past decade, several experimental studies on the behaviour of members strengthened with UHPC have been conducted. UHPC has been applied to existing RC members through either in-situ casting or the use of epoxy adhesive (in the case of prefabricated UHPC layers); after they have been strengthened, the members have been shown to achieve enhanced structural performance as composite members [4–11].

The interfacial bonding behaviour of composite UHPC–concrete members has been reported through experimental investigation [6–8,11]. Local failures such as the de-bonding and/or fracture of UHPC in the interface zone can cause the premature failure of composite members [7,8]. According to Habel et al. [11], the bond between NSC and UHPC is stronger than the tensile strength of NSC at the level of the longitudinal rebar of the RC members. Noshiravani and Brühwiler [7] have reported that the structural resistance of composite UHPC–concrete members depends on the bonding conditions in the interaction between the three materials in tension (longitudinal rebar, UHPC, and NSC). Denarié et al. [6] have shown that the bonding mechanism between NSC and UHPC is more efficient in terms of energy dissipation than the mechanism in the cracking of RC members. Al-Osta et al. [4] conducted tests and reported that no de-bonding was observed but horizontally fine cracks appeared near the interface. Yin et al. [8] used UHPC as patch material for repairing deteriorated concrete members. The strengthened specimens failed in flexure, and no de-bonding occurred. When they used UHPC overlaid onto soffits of RC members, all the specimens failed in shear, and de-bonding was observed after the tests. The investigations in these previous studies [4,6–8,11] demonstrate that the strength of the bond between NSC and UHPC is a crucial factor in the performance of the composite.

Studies on the behaviour of composite UHPC–concrete members using the finite element (FE) method, which is an effective tool for numerical simulations, have been very limited in the past

[4,10,12,13]. Sadouki et al. [12] conducted the two-dimensional (2D) FE modelling of the response of the composite UHPC–concrete beams tested by Noshiravani and Brühwiler [7]. The simulation results showed good agreement with the experimental results. Lampropoulos et al. [13], Al-Osta et al. [4], and Safdar et al. [10] predicted the behaviour of composite UHPC–concrete members using three-dimensional (3D) FE analysis. In the study by Lampropoulos et al. [13], the bond interface was modelled by adopting a friction coefficient of 1.5 and a cohesion of 1.9 MPa for a well-roughened substrate, but no experimental data for UHPC–concrete members were used to validate their model. The simulation of UHPC–concrete members was conducted and compared only to the results for beams strengthened with conventional RC layers. Al-Osta et al. [4] and Safdar et al. [10] conducted FE simulations of the behaviour of the composite members under the assumption of a perfect bond at the NSC–UHPC interface. This simplification might result in the overestimation of the ultimate capacity of the composite members. However, none of the above-mentioned studies on the FE method addressed the interfacial bond characteristics between NSC and UHPC in FE model or validated the model with test data for composite UHPC–concrete members.

The main objective of the present study was to develop a 3D FE model with the bond strength between UHPC and NSC taken into account for the prediction of the behaviour of composite UHPC–concrete members using the non-linear FE software LS-DYNA [14]. The remainder of this paper is organized as follows. First, the experimental data used to validate the FE model, which were previously obtained by the present authors [8] and other researchers, are described. Second, the FE model and the material models adopted for the non-composite RC and UHPC members are presented. Next, the model development process is presented. A concrete damage model in LS-DYNA was used for both NSC and UHPC. The non-composite analysis was developed in preparation for the analysis of composite members. To model composite UHPC–concrete members, three analysis cases were simulated and compared. In two of these cases, the bond strength was neglected by assuming a perfectly bonded or unbonded interface between the NSC and UHPC. The third case is the proposed method, and it employs a novel modelling technique using equivalent beam elements at the NSC–UHPC interface to consider the bonding characteristics for composite UHPC–concrete members. Finally, the simulation results are validated and discussed.

2. Description of specimens

This section briefly describes the experimental specimens used to validate the FE model developed in the present study. A total of 17 specimens consisting of non-composite members and composite UHPC–concrete members were used.

Nine slab specimens with various composite UHPC–concrete configurations were tested by Yin et al. [8] under the three-point loading system shown in Fig. 1(a). Two of them were non-composite members (Fig. 1(b)); specifically, slabs RE-0 and RE-100 were composed solely of NSC and UHPC, respectively. Details of the geometry and material properties of the non-composite specimens are given in Table 1. The remaining seven specimens were composite UHPC–concrete slabs designed to investigate the application of UHPC in the tension zone as additional UHPC overlays or as a patch material for rehabilitation of concrete structures (Fig. 1(c)). At the time the composite specimens were constructed, the NSC surface was roughened using a chisel and hammer before the UHPC was cast to create a good bond interface. Details of the geometry and material properties of the composite UHPC–concrete specimens are summarised in Table 2. Each slab was 1600 mm long with a clear span of 1200 mm. All slabs had five 12-mm-diameter high tensile steel bars at the top and bottom, except for two UHPC-overlay-strengthened slabs with five additional 10-mm diameter high-tensile-strength steel bars in the UHPC layer (slabs OV-25a and OV-50a). No transverse shear reinforcement was provided; however, to avoid anchorage failure at the end supports, three 6-mm-diameter mild steel links were installed.

In addition, for the non-composite members, eight additional numerical models of UHPC beams tested by Yang et al. [15] and Yoo et al. [16] were used, as listed in Table 1. It should be noted that Yang et al. [15] did not report the yield strength of the longitudinal rebar; therefore, the previously reported strength of 445 MPa [17] was assumed in the FE simulation.

3. Finite element modelling of non-composite members

3.1. Modelling and conditions

Section 3 describes the modelling method for the non-composite specimens shown in Table 1. In the present study, LS-DYNA [14] (version R8.0) was used to simulate the overall response of composite UHPC–concrete specimens. A full-scale 3D model was prepared for each specimen. NSC and UHPC were modelled using eight-node solid elements. For the longitudinal rebar in the specimens, two-node beam elements were used. A perfect bond was assumed between the longitudinal rebar and the concrete. A mesh size of 10 mm was used for the slab specimens previously tested by the present authors [8], and a mesh size of 20 mm was used for the beam specimens tested by Yang et al. [15] and Yoo et al. [16]. These mesh sizes were determined through convergence investigation to ensure the model can yield good predictions. The overall configuration of the FE model is shown in Fig. 2.

Support boundaries were modelled using simply supported conditions, as in the experiments. The nodes of the left support of each specimen were restricted in all translational directions, and those of the right support were allowed to move freely in the horizontal direction with respect to the longitudinal axis of the specimen. All nodes were free in the rotational directions.

Implicit analysis, which is suitable for static loading conditions, was used to numerically solve the iterative equation. A loading rate of 2×10^{-5} m/s was adopted for quasi-static loading. The load was applied directly to the nodes, and displacement-controlled loading was used. Because the experiments used to validate the FE model were performed under static conditions, the strain rate effect was not considered.

3.2. Concrete material model

The concrete damage model Mat-72r3 in LS-DYNA was used in the FE analysis to simulate the behaviour of both NSC and UHPC in the present study. The reliability of the concrete model Mat-72r3 has been previously reported [18,19]. This concrete model, also known as the Karagozian & Case (K&C) concrete model, was first developed for DYNA3D [20]. In Release III of the K&C model, an automatic input capability was added, and this model is currently available in LS-DYNA as material type 72r3. The major advantage of the model is that it requires a single parameter, the unconfined compressive concrete strength f'_c, as an input. The remaining parameters are automatically generated using a built-in algorithm and can also be modified by the user. A brief overview of the concrete damage model is presented in the following sections.

3.2.1. Strength surfaces

The concrete damage model is a plasticity-based constitutive model for concrete using three independent strength surfaces: an initial yield surface, a maximum failure surface, and a residual surface. The function of each of these strength surfaces can be expressed as [20]

$$\Delta\sigma_y = a_{0y} + \frac{p}{a_{1y} + a_{2y} p} \tag{1}$$

$$\Delta\sigma_m = a_{0m} + \frac{p}{a_{1m} + a_{2m} p} \tag{2}$$

$$\Delta\sigma_r = \frac{p}{a_{1r} + a_{2r} p} \tag{3}$$

where p is the pressure; and $\Delta\sigma_y$, $\Delta\sigma_m$, and $\Delta\sigma_r$ are the initial yield, maximum failure, and residual surfaces, respectively. The eight parameters (a_{0i}, a_{1i}, a_{2i}) define the three-parameter failure surfaces.

The failure surface $\Delta\sigma$ for the deviatoric stresses, which is based on the second invariant of the deviatoric stress, is defined as

$$\Delta\sigma = \sqrt{3J_2} \tag{4}$$

where $J_2 = (s_1^2 + s_2^2 + s_3^2)/2$ is the second invariant of the deviatoric stress; and s_1, s_2, and s_3 are the principal deviatoric stresses.

The plasticity surface representing the strain hardening after it reaches the yield surface is obtained as the interpolation between the initial yield and maximum failure surfaces, which is given by

$$\Delta\sigma = \eta\left(\Delta\sigma_m - \Delta\sigma_y\right) + \Delta\sigma_y \tag{5}$$

Similarly, the post-failure surface for strain softening is defined as the interpolation between the maximum failure and residual surfaces as

$$\Delta\sigma = \eta\left(\Delta\sigma_m - \Delta\sigma_r\right) + \Delta\sigma_r \tag{6}$$

The surface interpolation is accomplished by internally scaling the softening and hardening of the variable η, called the yield scale factor, which is determined from the damage function λ as

$$\lambda = \begin{cases} \displaystyle\int_0^{\overline{\varepsilon_p}} \frac{d\overline{\varepsilon_p}}{\left(1 + p/f_t\right)^{b_1}} & p \geq 0 \\ \displaystyle\int_0^{\overline{\varepsilon_p}} \frac{d\overline{\varepsilon_p}}{\left(1 + p/f_t\right)^{b_2}} & p < 0 \end{cases} \tag{7}$$

where $\overline{\varepsilon_p}$ is the effective plastic strain, b_1 and b_2 are the damage parameters for the concrete hardening and softening behaviour, respectively, and f_t is the quasi-static concrete tensile strength.

The value of η varies from 0 to 1 depending on the accumulated effective plastic strain parameter λ. A series of (η, λ) pairs was used as inputs in LS-DYNA. As shown in Fig. 3, η begins at 0 at $\lambda = 0$, increases to 1 at some damage function value $\lambda = \lambda_m$ (maximum), and then decreases to 0 at some larger value of λ. When $\lambda \leq \lambda_m$, the current surface is obtained as the interpolation between the initial yield and maximum failure surfaces. For $\lambda > \lambda_m$, the surface is obtained as the interpolation between the maximum and residual surfaces.

3.2.2. Equation of state

An equation of state (EOS) of the tabulated compaction was employed in the concrete damage model. With this EOS, which can capture volumetric hardening, the concrete damage model becomes simple and flexible for use in calibrating the model for UHPC [20]. The EOS is given as the relationship between the pressure P and volumetric strain ε_v as [14]

$$P = C(\varepsilon_v) + \gamma T(\varepsilon_v) E \tag{8}$$

where ε_v is the volumetric strain given by the natural logarithm, C and T are coefficients that are functions of ε_v, γ is the specific heat ratio, and E is the internal energy.

3.2.3. Determination of model parameters

As described in Section 3.2.1, the concrete damage model contains a number of parameters to express the concrete behaviour. According to Eqs. (1)–(3), the eight parameters (a_{0i}, a_{1i}, a_{2i}) should be determined through unconfined and triaxial compression tests over a range of confining pressures [20].

However, in the present study, the model parameters were not defined using the properties of actual concrete because of insufficient test data. In Release III of the concrete damage model (Mat-72r3 in LS-DYNA), two simulation methods are available. The first is a simple method using automatic parameter generation that requires only the concrete compressive strength f'_c as an input and calculates the other parameters as functions of f'_c. The second method requires detailed input parameters describing the concrete properties, posing the difficulty of requiring a variety of data obtained from laboratory tests as model inputs. In the present study, a combination of the first and second methods was adopted. The first method was preliminarily conducted by inputting f'_c to obtain all of the other parameters required in the second method. Then, the second method was carried out to modify the EOS. The initial stiffness of the members was adjusted based on the pressure and bulk modulus in the EOS to reflect the modification of Young's modulus of concrete. The pressure p calculated from the EOS is given as

$$p = K\varepsilon_v \tag{9}$$

where K is the loading or unloading bulk modulus and ε_v is the elastic volumetric strain.

The parameters b_1 and b_2 in Eq. (7) are employed in the concrete damage model to control the concrete hardening and softening behaviour. Although the details are omitted here, from an investigation of the effects of b_1 and b_2 in the model developed in the present study, similar to findings in [21], changes in the compressive softening parameter b_1 did not significantly affect the flexural performance of the specimen model, whereas changes in the tensile softening parameter b_2 demonstrated a clear effect. Based on this, the default value of b_1 (= 1.6) was used, and b_2 was modified from its default value (b_2 = 1.35) and set to −10 and −25 to reflect the actual behaviour of NSC and UHPC, respectively.

The localised crack width parameter w_c is used to eliminate the mesh size dependence in the concrete damage model. The tensile fracture energy under the stress–strain curve can be adjusted by varying w_c. The recommended value of w_c is three times the aggregate size for NSC [14]. For UHPC, steel fibres play a more important role in defining the tensile fracture behaviour than do the other fine constituents of UHPC. However, the optimal value of w_c for the FE model of UHPC remains unclear. In the present study, values of w_c = 25.18 mm (default) and 13 mm were adopted in the NSC and UHPC models, respectively. The parameters of the concrete damage model used for the specimens tested by the present authors are listed in Table 3.

3.3. Material model for longitudinal rebar

Material model type 3 (Mat-03) in LS-DYNA, which is an elastic–plastic model with kinematic and isotropic hardening, was used to model the longitudinal rebar in the present study. A Young's modulus of 200 GPa was adopted for this material. The slope of the bilinear stress–strain curves (tangent modulus) was assumed to be zero, representing perfectly constant stress after yielding. The properties used to model the longitudinal rebar were the same as those used in the authors' previous study [8] and are given in Table 3.

4. Modelling of composite UHPC–concrete members

4.1. Numerical model of perfectly bonded interface

Section 4 describes the modelling method for the composite UHPC–concrete specimens listed in Table 2. As previously mentioned, three analytical cases for the bond interface between NSC and UHPC were considered. Case 1 is the case of a perfectly bonded interface, which was considered to investigate the response of the specimens when UHPC is ideally bonded to the NSC substrate. Case 2 is the case of a perfectly unbonded interface, which was used for reference. Case 3 demonstrates the application of the newly proposed model using equivalent beam elements to consider the bond strength at the interface. The same parameters used for the non-composite members described in Section 3 were adopted for all three analytical cases.

In Case 1, a perfectly bonded interface between the NSC and UHCP of the composite UHPC–concrete specimens was assumed, and shared nodes were adopted at the interface. Except for the interface, the FE models used in Case 1 were the same as those described in Section 3. Fig. 4 illustrates the configuration of the FE model for Case 1. Details of the perfect bonding assumption are shown in Fig. 5(a) and (b).

4.2. Numerical model of unbonded interface

In Case 2, a perfectly unbonded interface between the NSC and UHPC of the composite UHPC–concrete specimens was adopted, and the nodes at the interface were not shared. Except for the interface, the FE models used in Case 2 were the same as those described in Section 3. To avoid element penetration between NSC and UHPC, the penalty-based automatic single-surface contact algorithm in LS-DYNA was employed at the interface. The modelling details of the unbonded interface are shown in Fig. 5(a) and (c).

4.3. Numerical model considering the bond strength at the interface

4.3.1. Modelling the bond interface

In Case 3, a modelling technique using equivalent beam elements at the NSC–UHPC interface was proposed to assess the interfacial bond strength for composite UHPC–concrete members. Instead of a friction or cohesive element at the interface [13], equivalent beam elements were adopted for stability in the FE simulation. The application of this modelling technique is simple. Equivalent beam elements were created along the longitudinal direction of the specimens through each node at the interface, and no transverse equivalent beam elements were provided. As in Case 2, Case 3 also used the automatic single-surface contact algorithm to prevent element penetration at the interface. The overall FE configuration with the proposed equivalent beam elements is shown in Fig. 6, and the modelling details are shown in Fig. 7. The FE model described in Section 3 was implemented to model the UHPC and NSC members. The equivalent beam elements were modelled using Mat-03 (Section 3.3) to provide elastic–plastic characteristics.

The interfacial bond between the NSC substrate and the UHPC can be expressed using the equivalent beam elements with an appropriate modelling technique for the bond behaviour at the interface. The nodes of the equivalent beam elements for the bond interface and those of the solid elements for the NSC and UHPC were intentionally created to coincide in order to enable node sharing at the

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interface. To avoid perfect bonding and consider a finite bond strength, the equivalent beam elements alternately shared nodes with the solid elements of the two types of concrete, as seen in Fig. 7(b). That is, when the *i*th node of the equivalent beam elements is shared with a node of the NSC solid elements, the (*i*+1)th node of the equivalent beam elements is shared with a node of the UHPC solid elements. The following sections focus on the characteristics of the interfacial bond between the NSC and UHPC used in the FE model.

4.3.2. Interfacial bond for composite UHPC–concrete members

Because the equivalent beam elements were used to define the bond characteristics at the NSC–UHPC interface in the present study, the Young's modulus (stiffness) and yield strength (bond strength) of the equivalent beam elements were defined as inputs for Mat-03 in LS-DYNA. These properties were assumed to correspond to those of the weak concrete NSC. The Young's modulus and yield strength were obtained using the following equations.

The stiffness K_c of NSC is defined as

$$K_c = \frac{G_c A_c}{t_{c,bond}} \tag{10}$$

where G_c is the shear modulus of NSC, A_c is the area of the concrete surface at the interface, and $t_{c,bond}$ is the thickness of the bonded surface. The thickness $t_{c,bond}$ was assumed to be 1 mm.

The shear modulus G_c is given by

$$G_c = \frac{E_c}{2(1+\nu_c)} \tag{11}$$

where E_c is the Young's modulus of NSC and ν_c is the Poison's ratio of NSC.

The stiffness K_{eb} of each equivalent beam element is defined as

$$K_{eb} = \frac{E_{eb} A_{eb}}{L_{eb}} \tag{12}$$

where E_{eb}, A_{eb}, and L_{eb} are the Young's modulus, area, and length of the equivalent beam element, respectively.

At the equivalent state ($K_c = K_{eb}$), E_{eb} is expressed as

$$E_{eb} = \frac{E_c}{2(1+\nu_c)} \frac{A_c}{A_{eb}} \frac{L_{eb}}{t_{c,bond}} \tag{13}$$

The yield strength of the equivalent beam elements was defined based on the equivalent bond strength of UHPC to NSC substrates. Because previous investigations on the bond strength of NSC to UHPC have been very limited in previous studies [2,3], a maximum bond strength of 0.55 MPa, as suggested by ACI 318 [22] for intentionally roughened surfaces, was adopted. The yield strength $f_{y,eb}$ was then defined as

$$f_{y,eb} = \frac{\tau_{\max} A_c}{A_{eb}} \tag{14}$$

where τ_{max} is the maximum bond strength (τ_{max} = 0.55 MPa [22]). The simplified elastic–plastic characteristic curve shown in Fig. 8 was adopted for the equivalent beam elements. The overall features of the bond can be found in [23,24].

5. Simulation results and verification

5.1. Results for non-composite specimens

Fig. 9 shows the simulated and experimental load–deflection curves for the two non-composite specimens RE-0 and RE-100 [8]. Good agreement was observed between the simulation and experimental results. For the NSC slab (RE-0), both the experimental and numerical curves showed a sudden drop in the force just after the peak load, as shown in Fig. 9(a). For the UHPC slab (RE-100), the FE simulation yielded a load–deflection curve with ductile behaviour that was very similar to the experimental results, as shown in Fig. 9(b). In addition, for both specimens, the simulated peak loads were approximately equal to the loads obtained experimentally, and the predicted maximum loads occurred at approximately the same deflections as in the experiment. Fig. 10 shows the damage pattern of the effective plastic strain distribution obtained from the numerical model along with the experimental final cracking pattern of the specimen RE-100. The numerical simulation showed a high effective plastic strain at the midspan, which agreed well with the experimental observation.

Figs. 11 and 12 show the experimental and simulated load–deflection curves for the non-composite specimens tested by Yang et al. [15] and Yoo et al. [16], respectively. It should be noted that only the key points of load–deflection curves, such as the first cracking load, yield load, peak load, and the corresponding midspan deflections, were reported by Yang et al. [15] for specimens R12-1, R22-2, and R23-2. These points were used for comparison and are depicted in Fig. 11. As shown in Figs. 11 and 12, the overall response of the load–deflection curves for the UHPC beams obtained from the FE simulation agreed well with the corresponding experimental curves.

5.2. Results for composite UHPC–concrete specimens

5.2.1. Results obtained from analysis of perfectly bonded interface

The load–deflection curves for the composite UHPC–concrete specimens obtained from the FE model under the assumption of perfect bonding at the NSC–UHPC interface (Case 1, Section 4.1) are illustrated in Fig. 13. From Fig. 13, when the interface between NSC and UHPC was perfectly bonded, the ultimate loads obtained from the numerical simulation were significantly higher than those obtained experimentally for all specimens except OV-25 and OV-50. It should be noted that specimens OV-25 and OV-50 had no longitudinal rebar in the UHPC overlay.

5.2.2. Results obtained from analysis of unbonded interface

With the assumption of a perfectly unbonded NSC–UHPC interface (Case 2, Section 4.2), the numerical estimates were below the experimental curves for all the specimens except RE-20, as shown in Fig. 13. From the results of Cases 1 and 2 in Fig. 13, when the effect of the bond strength is not taken into account in the FE model for the composite UHPC–concrete specimens, the analysis yields poor results, i.e., the simulation estimates curves that are significantly higher or lower than the experimental curves for perfectly bonded or unbonded interfaces, respectively.

5.2.3. Results obtained from proposed equivalent beam elements at the interface

The simulated load–deflection curves obtained from the FE model considering the bond strength effect using the proposed equivalent beam elements at the NSC–UHPC interface (Case 3, Section 4.3) were compared with the experimental results and those from analysis in Cases 1 and 2 (Fig. 13). As shown in Fig. 13, the curves obtained in Case 3 were between those obtained in Cases 1 and 2 and agreed well with the experimentally obtained curves. The FE model in Case 3 accurately predicted the initial stiffness and ultimate load for all of the composite specimens. For composite slabs RE-32 and RE-50, the simulated load–deflection curves of Case 3 were in good agreement with the experimental curves throughout the loading history, as shown in Fig. 13(b) and (c), respectively. For the other composite specimens, although the experimental results indicated that the specimens underwent ductile behaviour, the simulated curves showed a sudden drop after the peak load. These results may have been caused by the effect of the bond strength between the longitudinal rebar and concrete [8,25,26], as the proposed model does not consider the effect of the strength of this bond and instead assumes a perfect bond at this interface. However, the analysis in Case 3 demonstrated the effectiveness of the proposed method in the prediction of the initial stiffness and peak load and achieved fair agreement with the experimental results for the ultimate midspan deflections.

Figs. 14 and 15 show damage maps of the effective plastic strain obtained from the FE simulations along with sketches obtained from the experimental results of the final crack patterns for the composite slabs RE-20 and OV-25, respectively. These sketches show that the features of the effective plastic strain results from the analysis of Cases 1 and 3 were roughly similar to each other but quite different from the Case 2 analysis. This is because the shared nodes in Case 1 and the equivalent beam elements in Case 3 transferred the effective plastic strain to the UHPC layer, whereas the unbonded interface in Case 2 did not allow the transmission of any high effective plastic strain to the UHPC layer. The regions of high effective plastic strain in Figs. 14(c) and 15(c) for Case 3 were found to be fair agreement with the corresponding experimental cracking patterns in Figs. 14(d) and 15(d).

Fig. 16 shows the experimental-to-numerical peak load ratio P_{exp}/P_{FEM} obtained from the FE simulations in the cases of a perfectly bonded interface (Case 1), an unbonded interface (Case 2), and equivalent beam elements at the interface (Case 3) for the composite UHPC–concrete specimens. It was clearly demonstrated that the results in Case 3, which considers the bond strength at the NSC–UHPC interface, showed good accuracy in terms of the P_{exp}/P_{FEM} ratios, with the ratios in all cases approximately at the target line of P_{exp}/P_{FEM} = 1.0. In contrast, Cases 1 and 2 greatly deviated from the target line (i.e., yielded erroneous results).

5.3. Peak load and corresponding midspan deflection

The experimental and numerical peak loads P_{exp} and P_{FEM} and the corresponding midspan deflections Δ_{exp} and Δ_{FEM} were compared for all non-composite and composite specimens, where the P_{FEM} and Δ_{FEM} values were those obtained using the proposed equivalent beam element method (Case 3). The experimental-to-numerical peak load ratios P_{exp}/P_{FEM} and the corresponding peak load deflection ratios $\Delta_{exp}/\Delta_{FEM}$ were calculated for the Case 3 analysis, and the results are given in Table 4. From Table 4, the numerical peak loads showed good accuracy, achieving an average P_{exp}/P_{FEM} ratio of 1.01 and a coefficient of variation (COV) of 5.5%. The predicted corresponding midspan deflections also showed fair correlations, yielding an average $\Delta_{exp}/\Delta_{FEM}$ ratio of 1.13 and a COV of 31.2%.

Fig. 17(a) and (b) show the numerical results of the non-composite and composite (Case 3) specimens plotted against the experimental results for the peak load and the corresponding midspan deflection, respectively. As shown in Fig. 17(a), the peak load results showed a very good distribution along the target line representing $P_{FEM} = P_{exp}$. For the midspan deflection at the peak load shown in Fig. 17(b), although the results did not lie along the target line representing $\Delta_{exp} = \Delta_{FEM}$, most of the data points fell within the $\Delta_{exp} = \Delta_{FEM} \pm 20\%$ bounds, which indicates fair agreement.

6. Conclusions

The structural response of composite UHPC–concrete members was simulated using the developed FE model with LS-DYNA software. The model was based on the calibrated parameters of the concrete damage model obtained for non-composite members (NSC or UHPC specimens). For composite members, the proposed modelling technique using equivalent beam elements was adopted to represent the bond behaviour at the interface between the NSC and UHPC slabs. For comparison, FE analysis was also conducted assuming a perfectly bonded or unbonded interface. From the present study, the following conclusions could be drawn.

(1) The developed FE model yielded good predations for the overall response of both the non-composite and composite UHPC–concrete members.

(2) The proposed technique using the equivalent beam element at the interface between the NSC and UHPC was effective and efficient for simulating the behaviour of composite UHPC–concrete members. These equivalent beam elements characterised by appropriate concrete properties were able to adequately capture the bond performance.

(3) The proposed FE model for the composite UHPC–concrete members accurately predicted the load–deflection curves, whereas the curves from the analysis cases in which a perfectly bonded or unbonded interface were considered showed relatively poor correlation. In addition, the overall configuration of the effective plastic strain obtained from the proposed FE model roughly agreed with the crack damage patterns observed in the experiments.

(4) The experimental-to-numerical peak load ratio P_{exp}/P_{FEM} obtained using the equivalent beam elements showed good accuracy, with the ratio approaching the target line (P_{exp}/P_{FEM} = 1.0). In contrast, the perfectly bonded or unbonded interface analysis cases yielded a large deviation. However, the corresponding midspan deflection obtained from the proposed FE model showed only fair agreement with the experimental results.

The numerical investigation in the present study revealed that the bond strength between NSC and UHPC should be taken into account when conducting the FE analysis of composite UHPC–concrete members. The results of this investigation demonstrate that although UHPC exhibits a good bond quality with RC members, it would not form a perfectly bonded interface. However, in future work, the effect of the mechanical concrete zone covering the longitudinal rebar on the overall numerical response should also be considered, and the post-peak ductile behaviour of the model should be improved.

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